

Analysis of Four Axial Compression Load Tests on High Capacity Augered Cast-in-Place Piles in the Texas Gulf Coast Soils

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Abstract

Four 460-mm diameter augered cast-in-place piles were tested in static axial compression for two high-rise residential projects in Houston, Texas. Pile capacities were evaluated from the interpreted failure load determined from the load-displacement curve for each load test based on Davisson offset criteria. Test specific α values were calculated based on the analysis of side resistance data. Based on the results of load tests, higher α values were used in the design to calculate side resistance for the production piles. The load tests provided useful information on the load transfer behavior of soils present in Texas Gulf Coast, and confidence in the design and construction of augered cast-in-place piles. The augered cast-in-place piles proved to be more economical than other deep foundation systems commonly used in the Texas Gulf Coast soils.

Keywords: Augered cast-in-place piles, load tests, ultimate capacity, side resistance, load transfer, the α method, the β method

1 Introduction

Augered cast-in-place (ACIP) piles have been used in various infrastructure and commercial projects in the Texas Gulf Coast since the early 1990s. As proof tests, load tests are routinely recommended as part of design and construction of ACIP pile foundation systems. The side resistance and the pile tip bearing resistance are predicted based on available design methods including the FHWA method. Various researchers and investigators have developed methods to predict ultimate pile capacities. Popular among them are Bustamante and Gianselli (1981), FHWA method (Reese and O'Neill, 1988), Neely (1991), Viggiani (1993), McVay et al. (1994), Zalada and Stephenson (2000), Kulhawy et al. (2004), and Vipulanandan et al. (2005), etc. These methods need to be verified using full-scale field load tests to evaluate their appropriateness and efficacy as design tools in a particular geology.

Recent observations of soil failures at West Delta 58A in the Gulf of Mexico by Jardine and Saldivar (1999) provide new insight into a failure surface away from the pile-soil interface towards the native soil making the capacity prediction by these methods conservative. These methods are being routinely verified using full-scale load tests to more accurately predict pile capacity for design and construction of ACIP pile foundations in the Texas Gulf Coast.

2 Existing Design Methodologies

The general equation for computing the ultimate pile capacity in axial compression is given by:

$$Q_T = R_B + R_S \tag{1}$$

where, Q_T = ultimate/total pile capacity,

R_B = ultimate end bearing capacity, and
 R_S = ultimate side resistance.

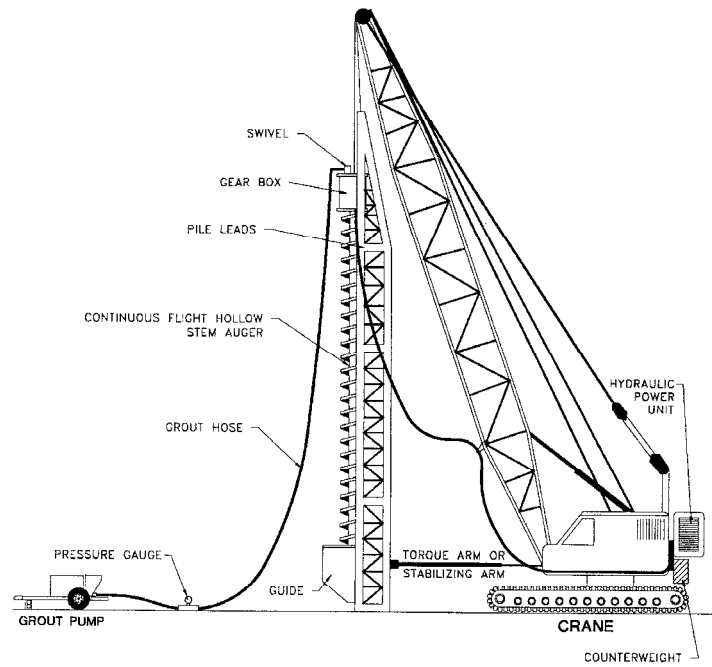


Figure 1. Schematic of ACIP pile installation equipment.

Both R_B and R_S are displacement dependent and develop their limiting values at significantly different displacements. At a displacement of about 5% of the pile diameter, both terms can be added. Moreover, the 25mm settlement limit often used by structural engineers is approximately 5% of a 460-mm diameter ACIP pile. Normally, the net ultimate pile capacity is computed without including the pile weight. Side resistance of ACIP piles with diameters up to 460 mm ranges from about 85% to 95% of total pile capacity in the Beaumont clays along the Texas Gulf Coast. For design purposes, the end bearing resistance in clays is neglected in the local practice due to the small tip area of the piles. Therefore, ACIP piles are designed as "friction" piles for diameters up to 460 mm.

Side resistance of ACIP piles is mobilized through unit interface shearing resistance, f_s , available along the effective pile surface and is given by the general equation:

$$R_S = \int_0^D f_{\max} P_z dz \quad (2)$$

where, f_{\max} = peak unit friction/side resistance,
 D = penetration depth of the pile, and
 P_z = the pile circumference at depth z .

For a pile having a diameter B , Equation 2 becomes:

$$R_S = \pi B \int_0^D f_{\max} dz \quad (3)$$

There are two methods available to compute f_{\max} for determining side resistance: the α method (total stress method) and the β method (effective stress method). There are various versions of these methods presented in the literature. Most of the ACIP pile design methods in clays are based on the use of undrained shear strength. O'Neill (2001) suggests that the α method is very practical in cohesive soils because the soil properties are easy to obtain accurately from laboratory testing and, therefore, are used widely for prediction of the pile capacities.

For the design of piles in clays, the peak value of unit side resistance is computed as:

$$f_{\max} = \alpha s_u \leq s_u \quad (4)$$

where, α = adhesion factor based on experimental results from full-scale load tests, and
 s_u = average undrained shear strength of clay along the effective pile length.

The FHWA method (Reese & O'Neill, 1988), Neely (1991), Viggiani (1993), McVay et al. (1994), Zalada and Stephenson (2000), Kulhawy et al. (2004), and Vipulanandan et al. (2005) suggest using the β method which predicts side resistance in sands based on the effective stress principles including coefficient of lateral earth pressure. According to McVay et al. (1994), the mobilized side resistance in sands at a given pile depth is given by:

$$f_s = p'_o K_s \tan \phi = \beta p'_o \leq 150 \text{ kPa} \quad (5)$$

where, p'_o = average effective vertical stress along the pile,
 K_s = earth pressure coefficient = 1.1, and
 β = friction parameter

In the FHWA method, the β value is a function of depth z . In contrast, Neeley's β parameter is a function of the pile depth. Lateral earth pressure coefficient, K_s , may vary approximately within 0.30 to 4.0 depending on stress history, and loading conditions mobilizing active, at-rest or passive earth pressures.

$$\beta = 1.5 - 0.1354z^{0.5} \quad 0.25 \leq \beta \leq 1.2 \quad (6)$$

O'Neill and Hassan (1994) suggest that the values of the α factor depend on the construction methods used for ACIP piles including the effects of drilling disturbance on the soil, roughness of the interface between grout and soil, pore water pressure changes during loading, and the test method to assess s_u . Generally, s_u is taken as one-half of the soil compression strength. Direct correlations in clays with strength measurements made by *in situ* tools such as the quasi-static cone penetrometer, pressuremeter, and dilatometer are not common in the US geotechnical engineering practice. Load transfer is neglected within the active zone based on the measurements by O'Neill and Reese (1972). For clays with $s_u \leq 96$ kPa, Hassan and O'Neill (1993) provided a limiting value of α of 0.55 as used in the FHWA method. The Texas Department of Transportation (TxDOT, 2000) uses a limiting α value of 0.70 to determine peak unit side resistance from the undrained shear strength of the clays. O'Neill et al. (2002) and Zelada and Stephenson (2000) concluded that ACIP piles in coarse-grained soils develop about the same compression capacities that are mobilized in drilled shafts using the β method. Dass (2003) and Dass and Puri (2006) observed higher α values to predict side resistance based on their analysis of drilled shaft load test data in Beaumont clays along the Texas Gulf Coast. Zelada and Stephenson (2000) provided a correction factor of 0.8 to β values determined using the relationship given by Reese and O'Neill (1988). Values of the β parameter range from 0.2 to 1.0. The β value is controlled by the lateral effective stress, stress history, installation procedure, and loading. A limiting unit side resistance of 96 kPa is used in design to predict side resistance of ACIP piles in Gulf Coast soils.

3 Load Test Program

Four full-scale load tests were conducted as part of the design and construction of foundation systems at two sites in Houston, Texas. The tests were performed under static axial compression using the Quick Load Test Method (ASTM D 1143). The maximum capacity of the reaction frame was 3560 kN. Davisson offset method (1972) was used to evaluate the ultimate capacity from the load-displacement curves. Four test piles were installed with pile tips bearing in clays based on site soil stratigraphy developed from soil borings.

3.1 Subsurface Stratigraphy

The predominant soil types along the Texas Gulf Coast are high plasticity, over-consolidated Beaumont clays and slightly over-consolidated sands (Al-Layla, 1970 and Mahar and O'Neill, 1983). Figure 2 shows the geology along the Texas Gulf Coast including the Beaumont Formation. Soils in this study included uniform clay strata and mixed soil profiles. The Beaumont clays are stiff to hard, desiccated clays having a network of closely spaced, random-oriented fissures with slickensides. The effective stress parameters of the Beaumont clays vary considerably depending on the predominant mineral and the soil structure.

The soil profile at the site for Load Test Nos. 1 and 2 was predominantly clays with lean clay and fat clay strata within the 30-m explored depth. The soil profile at the site for Load Test Nos. 3 and No. 4 was comprised of fat clay and lean clay strata

interbedded with sand strata. The undrained shear strengths of the clays encountered varied from 50 kPa to 280 kPa. The resistance expressed in corrected standard penetration test (SPT) blow counts per 305-mm penetration for sands encountered ranged from 10 to 55. In situ moisture contents of the soils varied from 8% to 27%. The liquid limits of the clays ranged from 30 to 70 with plasticity indices varying between 15 and 44. The dry density of the soils varied between 14 kN/m³ and 19 kN/m³. A preconsolidation pressure ranging from 200 kPa to 350 kPa was determined from the one-dimensional consolidation tests conducted on the clays.

3.2 Analysis of Load Test Results

Load-displacement curves for four tests are shown on Figures 3 through 6. The ultimate capacities were determined using the Davisson's offset. A pile tip resistance of 60 kN, based on the pile tip area for a 460-mm diameter pile and a limiting tip bearing resistance of 380 kPa according to the TxDOT design criteria, was deducted from the ultimate pile capacity to calculate the side resistance. Both α and β methods were used to predict pile side resistance for each test pile. Equations 4 and 5 were used with a limiting α value of 0.55 discussed in the FHWA method. The calculated β values for sand strata ranged from 0.5 to 1.1. Table 1 summarizes the results of four load tests and the corresponding back-calculated α values along with the predicted pile capacities using the FHWA method.



Figure 2. Geology of Texas Gulf Coast showing Beaumont Clay Formation.

3.3 Comparison

For calculation of the side resistance, the FHWA method developed by Reese and O'Neill (1988) is perhaps the most widely known and extensively used in design and construction of deep foundations and, therefore, was used in the study for comparison. Table 1 summarizes the pile capacities calculated using the FHWA method. The calculated α values produced about 40% greater unit side resistance than the unit side resistance obtained by the FHWA method. Allowable design pile capacity was calculated from the ultimate capacity using factors of safety of 2 and 3 for axial compression and uplift, respectively.

Table 1. Measured and predicted ultimate pile capacities with the calculated α values.

Pile ID	Pile Diameter (mm)	Pile Length (m)	Ultimate Test Capacity Q_t (kN)	Calculated α values	FHWA Q_t (kN)	$[Q_t]_M / [Q_t]_P$	Side Resistance $[Q_s]_M / [Q_s]_P$
1	460	22.90	2940	0.79	2040	1.44	1.45
2	460	22.90	3070	0.84	2040	1.50	1.52
3	460	25.9	3110*	0.75	2225	1.40	1.40
4	460	21.30	2620	0.72	2020	1.30	1.31

*Test stopped at design ultimate pile capacity and 88% of maximum capacity of the reaction frame.

$[Q_t]_M$ = Measured ultimate pile capacity from load-displacement plot.

$[Q_t]_P$ = Predicted ultimate pile capacity using the α method and the β method.

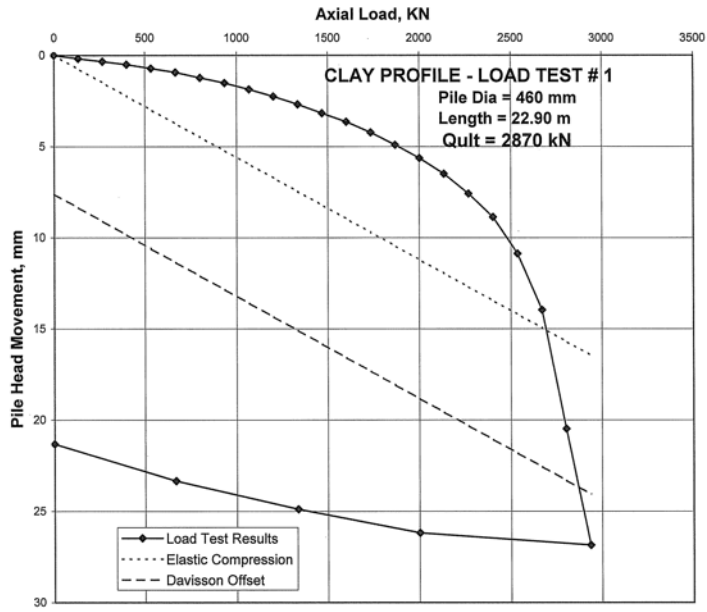


Figure 3. Load-Displacement curve for Load Test No. 1.

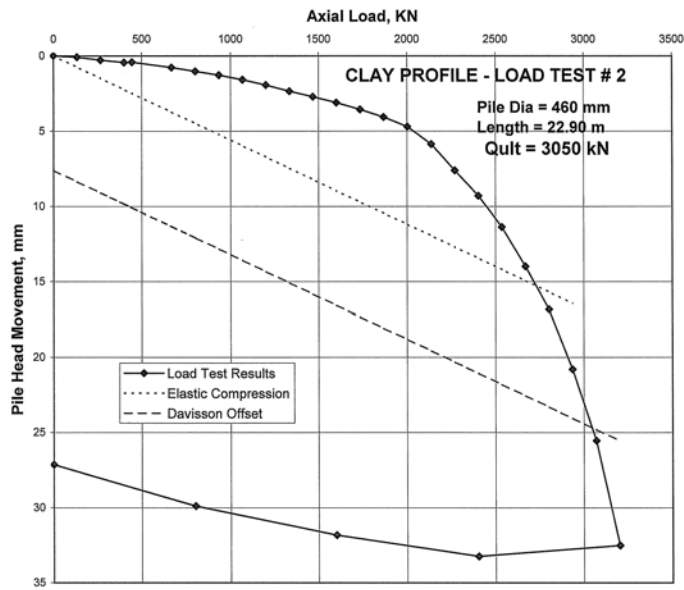


Figure 4. Load-Displacement curve for Load Test No. 2.

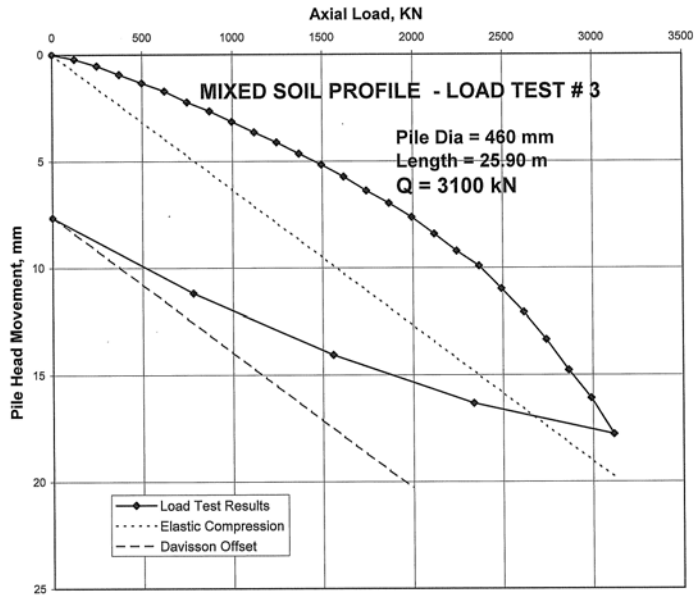


Figure 5. Load-Displacement curve for Load Test No. 3.

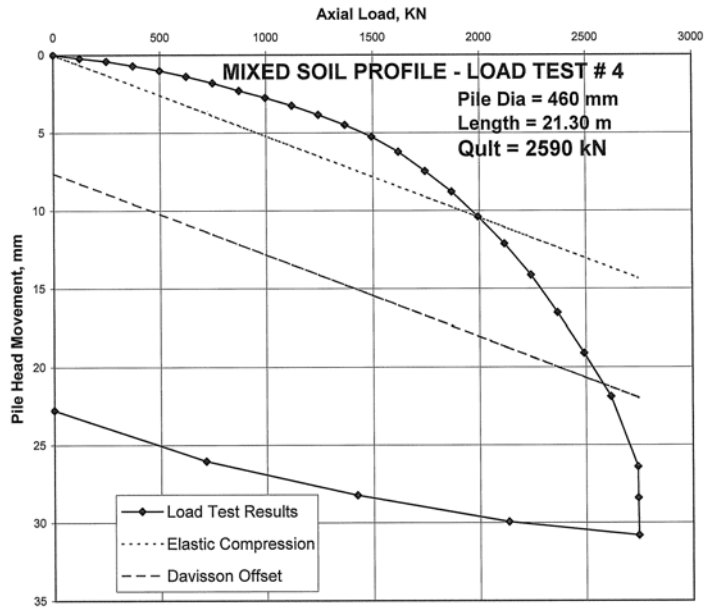


Figure 6. Load-Displacement curve for Load Test No. 4.

4 Conclusions

Based on the study, the following conclusions can be reached:

1. Side resistance of ACIP piles in over-consolidated Beaumont clays is being conservatively predicted using the existing methods.
2. Based on load test data for these two projects, α values of 0.72 to 0.84 with an average value of 0.78 were calculated for ACIP piles in Beaumont clays.
3. No appreciable change was noticed in the β values for sands encountered at the site.
4. Design of ACIP piles in Beaumont clays may be based on an α value of 0.75.
5. Load tests should be performed to confirm design capacity of ACIP piles in Beaumont clays for a more rational ultimate capacity resulting in higher pile capacities and lower foundation costs.

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